

Non synchronous earthquake motion in bridges design

S. Biondi

University of Chieti-Pescara, Italy

G. Leoni

University of Camerino, Italy

C. Nuti

University of Rome 3, Italy

G. Tropeano

University of Naples Federico II, Italy

I. Vanzi

University of Chieti-Pescara, Italy



SUMMARY:

The study aims to further develop, with respect to previous findings, and validate structural design criteria which account for the effects of earthquakes spatial variability. In past works [Nuti, C. and Vanzi, I. (2004) & (2005); Carnevale, L. et al. (2010)] the two simplest forms of this problem were dealt with: differential displacements between two points belonging to the soil or to two single degree of freedom structures. Existing codes appear indeed improvable on this aspect. For the differential displacements of two points on the ground, these results are generalized with different response spectra and validated using (indeed a small set of) real recordings. For the experimental validation, the first obtained results point towards an acceptable agreement of model vs. experimental results [Tropeano, G. et al. (2011)]. In any case, results indicate that the design codes can be improved on this topic, both for the two points (e.g. simply supported decks) and the multiple points (e.g. continuous decks on multiple piers) cases.

Keywords: Bridge design, Earthquake, Non synchronous motion, Support design, Random field, Probability

1. INTRODUCTION

Some different models defining the spatial variability of earthquakes have been developed in the last twenty years, departing from experimental observations of simultaneous recordings of earthquakes [Abrahamson, N.A. et al. (1991); Oliveira, C.S. et al. (1991)]. From the classical work of Luco and Wong [Luco, J.E. and Wong, H.L. (1986)], different statistical descriptions have been proposed and fit to the experimental data [Vanmarcke, E. H. and Fenton, G.A. (1991); Santa-Cruz, S. et al. (2000)], with varying degree of complexity and accuracy. The effects on structures have been also investigated, either in the linear field, with random vibration tools [Der Kiureghian, A. and Neuenhofer, A. (1991) & (1992)], or in the non linear one, via numerical simulations or equivalent linearization procedures [Monti, G. et al., (1994)&(1996); Hao, H. (1998); Sextos, A.G. et al. (2003)]. The most important outcome of the studies could be appearing definitive and unambiguous: apart from a few cases, non synchronous action decreases the structural stresses with respect to the case with synchronous actions. There are however situations in which non-synchronism negatively influences structural behavior, e.g. deck unseating and some of the current design rules provided by the Codes appear improvable on this aspect. This topic was deeply discussed by the Authors above all considering Code provisions refinement in last years. Departing from these observations on non-synchronism influence on structural response and considering results of previous studies, this paper aims to validate structural design rules which account for effects of earthquakes spatial variability: in particular two different Code provisions, according to Code changing, are considered and discussed.

In previous works [Nuti, C. and Vanzi, I. (2004) & (2005)] the two simplest forms of this problem were dealt with differential displacements between two points belonging to the soil or to two single degree of freedom structures. In these works seismic action was defined according to both EC8

[Comité Européen de Normalisation (2004); at the time assumed in an original Italian Code, Presidenza del Consiglio dei Ministri (2003)] and draft [at the time; now it was issued (Ministero Infrastrutture, 2008)] new Italian Code; the structures were assumed as linear elastic sdof oscillators. In recent papers [Biondi, S. et al. (2011), Carnevale, L. et al. (2010)] the previous results, in terms of differential displacements of two points, were validated and generalized using the newly developed response spectra contained in the new seismic Italian Code. Furthermore the problem of statistically defining the differential displacement among any number of points (which is needed for continuous deck bridges) is approached too and some preliminary results will be shown in this paper. The results of these approaches are univocal and different at the same time: current Codes (both EC8 and Italian Code 2008) may be improved on this aspect yet the Italian Code is more efficient.

2. THEORETICAL APPROACH

For the sake of completeness, a short summary of the model is presented herein; obviously readers are referred to previous works [Nuti, C. and Vanzi, I. (2004) & (2005)] for a more detailed presentation of mathematical aspects. An earthquake acceleration recording at point P in space can be represented, via its Fourier expansion, as a sum of sinusoids, Vanmarcke, E.H. and Fenton, G.A. (1991):

$$A_P(t) = \sum_k [B_{Pk} \cdot \cos(\omega_k \cdot t) + C_{Pk} \cdot \sin(\omega_k \cdot t)] \quad (1)$$

In equation (1), $A_P(t)$ is measured acceleration in point P at time t , k is an index varying from 1 to the number of circular frequencies ω_k considered, B_{Pk} and C_{Pk} are the amplitudes of the k^{th} cosine and sine functions. Assuming that the acceleration $A_P(t)$ is produced by a wave, in the ground, moving with velocity V it is possible to define the acceleration in any point of the surrounding space. Considering a different point in space, say Q , at distance X_{PQ} from P , in this point Q , at time t , the earthquake acceleration, depending on time delay τ_{PQ} of the signal, could be defined as:

$$A_Q(t) = \sum_k \{B_{Qk} \cdot \cos[\omega_k \cdot (t - \tau_{PQ})] + C_{Qk} \cdot \sin[\omega_k \cdot (t - \tau_{PQ})]\} \quad (2)$$

$$\tau_{PQ} = \frac{X_{PQ}}{V} = X_{PQ} \cdot \left(\frac{\cos(\psi)}{v_{app}} \right) \quad (3)$$

In equation (3) ψ is the angle between the vector of surface wave propagation and the vector that goes from P to Q and v_{app} is the surface wave velocity. Equation (1) and (2) are equal and acceleration amplitude depends on coefficients of Fourier expansion of sinusoids sum, in particular the amplitudes B_{Qk} and C_{Qk} would be respectively equal to B_{Pk} and C_{Pk} if the medium through which the waves travel did not distort them. But it isn't the case of a real medium; in this case B_{Pk} is correlated with B_{Qk} and C_{Pk} is correlated with C_{Qk} while the B 's and C 's are independent. I.e. the amplitudes B_{Pk} and C_{Qk} are statistically independent, for any points P and Q , and any circular frequency ω_k , with the only exception of B_{Pk} and B_{Qk} i.e. same circular frequency but different points in space. The same holds for C_{Pk} and C_{Qk} . In order to simplify the approach, some hypothesis could be done: in particular the amplitudes are assumed normally distributed with zero mean and this assumption is experimentally verified. With this assumption, in order to quantify the acceleration time histories in different points in space, equations (1) ÷ (3), all it is needed is the definition of the correlation between amplitudes and of their dispersion, as measured by the variance or, equivalently, of the covariance matrix of the amplitudes. The covariance matrix Σ of the amplitudes B and C is assembled via independent definition, at each circular frequency ω , of its diagonal terms (the variances in each space point and frequency) and of the correlation coefficients. The diagonal terms Σ_{PP} are quantified via a power spectrum; a traditional choice is the Kanai-Tajimi power spectrum, modified by Clough and Penzien [Clough, R.W. and Penzien, J. (1975)]:

$$\Sigma_{PP} = G_{PP}(\omega) \cdot d\omega \quad (4)$$

$$G_{PP}(\omega) = G_0 \cdot \frac{\omega_f^4 + 4 \cdot \beta_f^2 \cdot \omega_f^4 \cdot \omega^2}{(\omega_f^2 - \omega^2)^2 + 4 \cdot \beta_f^2 \cdot \omega_f^4 \cdot \omega^2} \cdot \frac{\omega^4}{(\omega_g^2 - \omega^2)^2 + 4 \cdot \beta_g^2 \cdot \omega_g^4 \cdot \omega^2} \quad (5)$$

where its parameters are the scale factor G_0 , the central frequencies of the filters, ω_f and ω_g , and their damping, β_f and β_g (see details in Nuti, C. and Vanzi, I. (2004)). The Kanai-Tajimi power spectrum was adopted in the previous papers and the correlation coefficient between the amplitudes was expressed via the coherency function:

$$\rho = \exp\left(-\omega^2 \cdot X^2 \cdot \left(\frac{\alpha}{\nu}\right)^2\right) \quad (6)$$

using the form originally proposed by Uscinski [Uscinski, B.J. (1977)] on theoretical grounds and Luco and Mita proposal [Luco, J.E. and Mita, A. (1987)]. The correlation decreases with increasing distance X and circular frequency ω and increases with increasing soil mechanical and geometric properties as measured by the ratio ν/α where α is the incoherence parameter, ν the shear wave velocity. The incoherence parameter α is the most difficult aspect in the coherency function assessment. For a more detailed discussion the reader is referred to previous paper [Nuti, C. and Vanzi, I. (2004)]; however, values in a range as wide as 0.02÷0.50 are reported in past experimental studies. Departing from the above earthquake spatial model, using random vibration concepts, it may be shown that the distribution of the maximum differential displacement can be found with the peak factor formulation [Vanmarcke, E.H. et al. (1999)] by setting:

$$Z_{s,p}^* = \sigma_{Z^*} \cdot r_{s,p} \quad (7)$$

where $Z_{s,p}^*$ is the displacement value which is not exceeded with probability p during an earthquake of duration s , and σ_{Z^*} is the standard deviation of Z^* . Typical values of the peak factor $r_{s,p}$ lie within 1.20÷3.50 range; $r_{s,p}$ is computed as set out in Vanmarcke, E.H. et al. (1999), in which proper account is taken for the non-stationarity of the response via the use of the equivalent damping.

3. DIFFERENTIAL DISPLACEMENTS BETWEEN TWO POINTS ON THE SOIL: CODE PROVISIONS VS. PREVIOUS AND CURRENT FINDINGS

In this chapter a comparison is made between some of the Code provisions and the findings of past and new analyses by the Authors. Only the case of differential displacement between two points on the ground is considered. In more detail, the Codes considered are:

- the European Seismic Code EC8 [Comité Européen de Normalisation (2004)], partially adopted by the Italian Seismic Code of 2003 [Presidenza del Consiglio dei Ministri (2003)]; this Code will be referred to with EC8/ICPC, meaning EuroCode 8/Italian Civil Protection Code.
- the new Italian Seismic Code [Ministero Infrastrutture (2008)]. This Code will be referred to as ICB, meaning Italian Code for Bridges. This Code, for non synchronism, has been drafted following also the results of Authors previous works [Nuti, C. and Vanzi, I. (2004) & (2005)].

The analyses presented are:

- a summary of the results obtained by the Authors using EC8/ICPC response spectra with soils type A, B, D (respectively rock, stiff soil, loose soil)
- some results obtained using the ICB for soil types A & B (corresponding to EC8 soil types A & B).

For both Codes, reference is made to the ultimate limit state. For this limit state, the Codes state the ground differential displacements be computed as in (8) & (9) with X_{PQ} distance between points P and Q , ε_P and ε_Q soil coefficients in P and Q , pga peak ground acceleration, $\{T_{PC}; T_{PD}\}$ and $\{T_{QC}; T_{QD}\}$ periods defining the response spectra in P and Q , ν_{app} surface and ν shear wave velocities. It is to note that EC8/ICPC, for the case of different P and Q soils, state the differential displacement be computed, using (8), as half times the square root of the sum of squares of the differential displacements on homogeneous soils.

$$\begin{cases} u_{PQ}^I = X_{PQ} \cdot pga \cdot \frac{T_C}{v_{app}} \cdot \left(\varepsilon \cdot \frac{1}{2 \cdot \pi} \right) \leq u_{PQ}^{I \text{ MAX}} \\ u_{PQ}^{I \text{ MAX}} = 0.025 \cdot pga \cdot \sqrt{(\varepsilon_P \cdot T_{PC} \cdot T_{PD})^2 + (\varepsilon_Q \cdot T_{QC} \cdot T_{QD})^2} \end{cases} \quad EC8/ICPC \quad (8)$$

$$\begin{cases} u_{PQ}^{II} = u_{PQ}^{II \text{ MIN}} + (u_{PQ}^{II \text{ MAX}} - u_{PQ}^{II \text{ MIN}}) \cdot \left\{ 1 - \exp[-1.25 \cdot (X_{PQ}/v)^{0.70}] \right\} \\ u_{PQ}^{II \text{ MIN}} = 1.25 \cdot 0.025 \cdot pga \cdot \left| (\varepsilon_P \cdot T_{PC} \cdot T_{PD}) - (\varepsilon_Q \cdot T_{QC} \cdot T_{QD}) \right| \\ u_{PQ}^{II \text{ MAX}} = 1.25 \cdot 0.025 \cdot pga \cdot \sqrt{(\varepsilon_P \cdot T_{PC} \cdot T_{PD})^2 + (\varepsilon_Q \cdot T_{QC} \cdot T_{QD})^2} \end{cases} \quad ICB \quad (9)$$

In all the analyses, the most severe condition for non synchronism, i.e. highest uncorrelation, has been studied; therefore the incoherence parameter, in equation (6), has been taken as $\alpha = 0.50$. In Figure 1 the response spectra of EC8/ICPC and the ICB are shown. A few words, compatibly with the sake of brevity and space, about the ICB spectra are convenient. The spectra, obviously, are defined by nearly the same relationships as the EC8, with three important exceptions: the maximum spectral acceleration amplification is soil and site dependent, the periods defining each interval of the spectrum (T_B & T_C , lower and upper corner limits of constant spectral acceleration branch and T_D corner limit between constant velocity and constant displacement ranges) depend on the soil type and on the maximum site spectral velocity and, finally, topographic effects are explicitly accounted for. In order to make a comparison between the model results obtained with the EC8/ICPC spectra (Figure 1 left), and those of the ICB (Figure 1 right), the above dependencies have been drastically simplified: the minimum value of the topographic effect (i.e. multiplicative parameter for topography = 1) has been adopted.

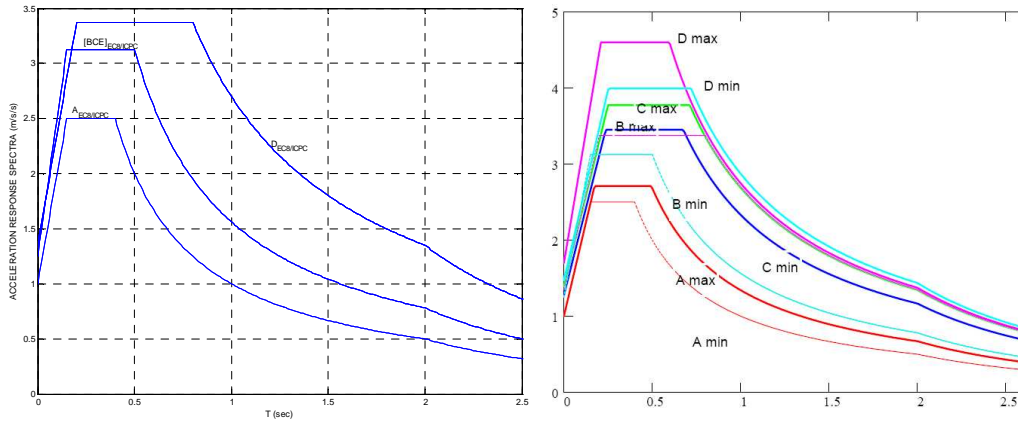


Figure 1. Acceleration response spectra of EC8/ICPC (left) and the ICB (right); $pga = 0.10 \text{ g}$. The min and max suffixes in the ICB spectra are relative to minimum and maximum topographic effects

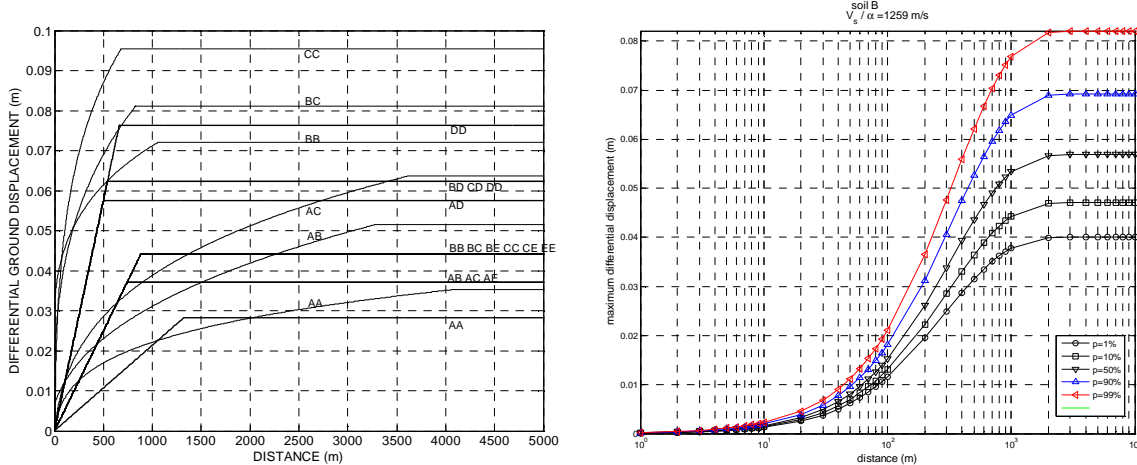


Figure 2. Left: soil differential displacements; thicker lines for EC8/ICPC; remaining lines for theoretical model. Right: differential displacement on soil type B for ICB spectra $pga = 0.10 \text{ g}$

Further, the maximum spectral velocity and maximum spectral acceleration amplification have been assumed constant and equal to the median values computed by Newmark and Hall [Newmark, N.M. and Hall, W.J. (1982)] for rock soil. These values are: PGV/PGA = 0.91 [(m/sec)/g]; maximum spectral acceleration amplification equal to 2.12·PGA; maximum spectral velocity equal to 1.65·PGV. With these hypotheses, both EC8/ICPC and ICB spectra depend only on the ground type and the peak ground acceleration. The first result is shown in Figure 2 (left). The figure shows the comparison between the soil differential displacements of EC8/ICPC versus those computed using the above discussed model, equation (7), now assumed in ICB. Notice that the results coming from the analysis shortly described have been cast in the form expressed by equations (9) for inclusion in the ICB. Examining Figure 2 (left), one can see that the maxima differential displacements computed with EC8 and this model differ by about 1.25; further, the trend is very different. EC8/ICPC increases linearly up to the maximum, the analyses results (and the ICB prescriptions too) grow in a parabolic fashion.

In the range of distances where most civil engineering structures are, between 5 and 100 m, from building columns to long bridges piers, the differences are large: at 20 m distance, EC8/ICPC gives 2 mm or less while ICB forecasts differential displacements from 2 mm to about 40 mm, depending on the soil coupling. The relative displacements computed with the ICB spectra for soil B, and with the Newmark and Hall simplification described before (in terms of maximum PGV and maximum PGA), are next shown in Figure 2, right. From Figure 2, one can notice that the increase of differential displacement with the distance is the same (the abscissa of Figure 2, left, are in natural scale while that of Figure 2, right, in logarithmic scale). The maximum values of differential displacements appear to indicate the dependence on spectral shape: with B soil type, maximum (at high distance) differential displacement is equal to 45, 72 and 58 mm respectively for EC8/ICPC, ICB (Figure 2 left) and ICB with Newmark and Hall hypotheses (Figure 2 right). These results indicate that there is indeed a dependence of the differential displacements on the spectral shape, although it must be investigated which part of the spectra this is due to.

4. DIFFERENTIAL DISPLACEMENTS BETWEEN ALIGNED POINTS ON THE SOIL: CODE PROVISIONS VS. PREVIOUS AND CURRENT FINDINGS

Bridges on multiple supports must be checked for spatial variability of seismic action. According to EC8 two different sets have to be considered; the first consists of relative displacements applied simultaneously with the same sign to all supports of the bridge in a considered horizontal direction. The second considers the case of ground displacements occurring in opposite directions at adjacent piers; for this latter case the displacement set, occurring at the base of the piers, is pictured in Figure 3. The displacement set consists in opposite direction displacements of the same value $d_i = \pm \Delta d_i / 2$; the relative displacement between two adjacent piers equals the maximum differential displacement $u_{PQ}^{II MAX}$ (see equation (9)) times the ratio between the average piers distance $L_{i,av} = (L_{i,i+1} + L_{i,i-1})/2$ and the distance beyond which ground motion may be considered uncorrelated, L_g , ranging from 600 m (soil A) to 300 m (soil D). In equation (10) β_r is a factor accounting for the magnitude of ground displacements occurring in opposite directions at adjacent supports; for different ground types this factor could be assumed as $\beta_r = 1$, this assumption is made in this paper for each type of ground.

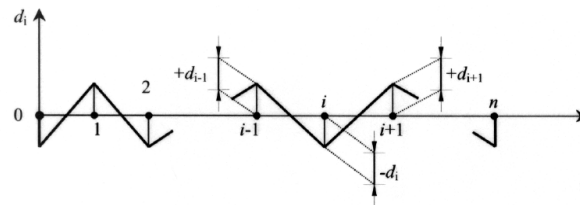


Figure 3. Displacement set for verification of multiple support bridges for displacements occurring in opposite directions (EC8/ICPC)

$$\Delta d_i = \beta_r u_{PQ}^{II MAX} \frac{L_{i,av}}{L_g} \quad (10)$$

For example, on soil type D, with average piers distance $L_{i,av} = 30$ m, $pga = 0.10$ g, the maximum differential displacement is equal to 78 mm (see Figure 2 left), so the relative displacement can be calculated as $\Delta d_i = \beta_r u_{PQ}^{II MAX} \frac{L_{i,av}}{L_g} = 1.00 \cdot 78 \cdot \frac{30}{300} = 7.80$ mm. This rule appears unconservative on one side (i.e. 7.8 mm appears too small a value) and far too conservative on the other (the probability that all the piers are displaced in opposite directions by the same amount is zero, obviously from an engineering view – point). Some preliminary analyses have then been carried out via Montecarlo sampling of the earthquakes generated with the model shortly described in the previous chapter. Three soil types, A, B and D, as defined by EC8/ICPC, have been assumed; the peak ground acceleration has been taken $pga = 0.10$ g while different piers distances are considered. The results of this analysis are shown in Figure 4 for soil type D.

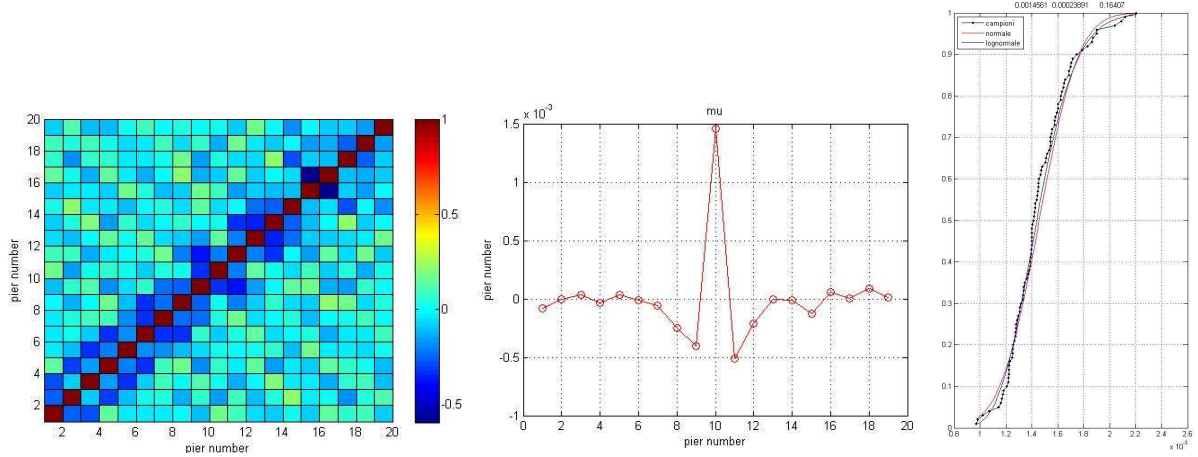


Figure 4. Statistics of soil curvatures and displacements for 21 piers at 20 m distance. 30 earthquake samples; soil D of EC8; $pga = 0.10$ g. Left: correlation; middle: mean value; right: cumulative distribution function

The statistics of soil curvatures, sampled at the base of the piers, show negative correlation (equal or higher than -0.5) between adjacent piers and no significant correlation thereafter (Figure 4, left). The statistics of curvatures may be therefore easily computed since those for two adjacent piers suffice to define the entire curvature field. The mean value (across the earthquake samples) of the maxima of curvatures is shown as the middle figure in Figure 4. The maxima are equal to $1.5 \cdot 10^{-3}$ (soil D) while lower values (not shown here) are $1.5 \cdot 10^{-4}$ (soil A, ten times lower) and $3.0 \cdot 10^{-4}$ (soil B, five times lower). The cumulative distribution function of the maxima of curvatures is finally shown as the right figure in Figure 4. Three curves are plotted: the sampled cdf and the normal (red) and lognormal (continuous blue one) interpolation. One can see that both approximations work rather well and that the three curves are rather undistinguishable.

Taking for simplicity the normal approximation as the reference one, the coefficient of variation of curvatures is approximately equal to 0.20 for all soil types; more precisely, it is equal to 0.20 (soil A), 0.22 (soil B), 0.16 (soil D). Hence, it appears reasonably simple to both define the mean values of the maxima of curvatures and the cdf of the maxima, for all soil types tested. One may sum up the obtained results as follows:

- the statistics of soil curvatures, sampled at the base of the piers, show negative correlation (about -0.5) between adjacent piers and no significant correlation thereafter,
- the statistics of curvatures should therefore be easy to compute since those for two adjacent piers suffice to define the entire curvature field,
- design should be done with the following soil relative displacements:
 - in i : $d_i = u_{PQ}(X_{PQ})$ (see equation (9))
 - in $i - 1$ and $i + 1$: $d_{i-1} = d_{i+1} = u_{PQ}/2$
 - elsewhere: 0
- the above values are the mean, across the earthquake sample, of maxima. The distribution of maxima can be modelled as a normal random variable with 0.20 c.o.v..

5. EXPERIMENTAL VALIDATION OF THE TWO POINT CASE AND RESPONSE OF CONTINUOUS DECK BRIDGES: FIRST RESULTS

This section documents the work (currently in progress) towards extension of the analyses. It contains first some remarks on the extension of the analyses to model the straight continuous deck bridge case. In the second subsection, a synthesis of the findings in Tropeano, G. et al. (2011), aimed at the validation of the two points case (section 3 of this paper) is illustrated.

5.1 Response of straight continuous deck bridges

The analyses to assess the structural response of continuous deck bridges is currently in progress in the ambit of the National Research Program dealing with “Bridges under non synchronous earthquakes: modelling, analysis and synthesis of the results” that was funded by the Italian Instruction Ministry [Nuti, C. (Coordinator) (2010)]. Some preliminary results of this activity are shown in a previous paper [Carnelvale, L. et al. (2010)] and will be summarized in this paper. The analyses documented in this paper are elastic ones and the bridges considered have six identical piers. The analyses were divided in two phases.

Firstly we have done some static analyses of the bridges with the non synchronous signals, in order to assess the main controlling variables; after elastic dynamic analyses were carried out, in order to control the structural response. In particular these latter have the aim of assessing the correctness of designing [as is currently done, equation (11)] for non synchronism via summation of the effects of fixed base response spectrum analyses plus static superimposed displacements at the piers base.

$$Z_M(t) = Z_P(t) + Z_{MP}(t) \quad (11)$$

where the total displacement in M , $Z_M(t)$ is the sum of the ground displacement $Z_P(t)$ and of the s.d.o.f. system displacement with respect to the ground $Z_{MP}(t)$. The response variables considered is one for the piers (the maximum top drift, denoted by δ) and two for the deck (the maximum bending moment and maximum shear, denoted by M and S). All the results discussed in what follows are the mean values of the response variables. Notice that we have consistently found the coefficient of variation to be between 0.1 and 0.2.

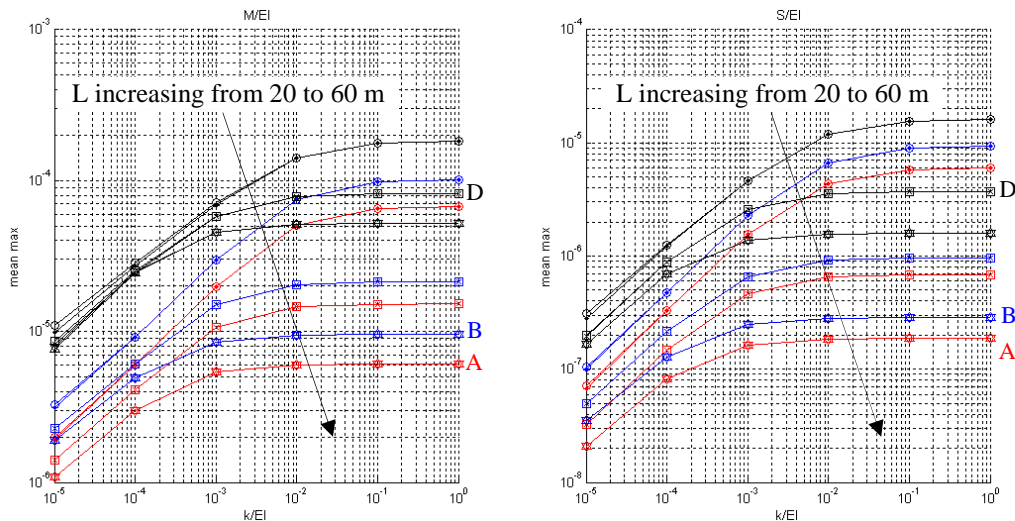


Figure 5. bending moment and shear force in the deck as a function of K/EI . $pga = 0.10g$

Besides, we denote the horizontal pier stiffness, deck flexural stiffness and length between two piers respectively with K , EI , L . First, the static analyses are discussed. Theoretically, it can be shown that maximum top drift of a pier depends on both pier stiffness and ratio between deck flexural stiffness and length between two piers; i.e. $\delta = \delta(KL^3/EI)$.

With 50 non synchronous earthquakes, sampled on soil types A, B, D, considering L varying between 20 and 60 m, K/EI varying between 10^{-5} and 10^0 (in MKS units), we have assessed the influence on the mean responses of K/EI and L . The results appear to prove the theoretical dependencies $\delta(KL^3/EI)$ i.e. that the pier drift depends solely on this stiffness ratio. The same dependency can be stated regarding to maximum bending moment of the deck $M/EI = (K/EI)L$ and maximum shear of the deck $S/EI = (K/EI)L$; these results are shown in Figure 5 and seem to prove the dependency of deck curvature on K/EI and L . Figure 5 diagrams give the correlations between the response and the input variables and they have permitted to calibrate the variables to investigate in the dynamic analyses.

The dynamic analyses are discussed in detail in [Carnelvale, L. et al. (2010)]. In this case the same input variables as the static analyses are used with the exception of length L , taken equal to 60 m, and the soil type, assumed of type A. The aim of those analyses was to check whether the dynamic bridge response (in terms of δ , M/EI , S/EI) to non synchronous earthquakes could be computed as the sum of a fixed base response spectrum analyses plus static superimposed displacements at the piers base. The preliminary results are discussed in Carnelvale, L. et al. (2010), in particular in terms of the correlation between the assumed K/EI and the period of the first bridge mode. The maximum shear and bending moment (for the deck) are depicted as a function of the bridge first natural period. For sake of simplicity and in order to control design procedure, all quantities are adimensionalised to the target response, i.e. the one computed with the dynamic non synchronous analyses. From the results it appears that, for the deck response variables, response spectrum values underestimate substantially the target results, while the displacement sum proves better.

On the contrary for the pier response variable, response spectrum analyses generally overestimate the target results and so do the displacement sum results. So, it generally appears that further research is necessary in order to define a simple and accurate design rule, above all regarding structural analysis; but, however, what is currently recommended in EC8 with ground displacements occurring in opposite directions at adjacent piers is certainly improvable.

5.2 Comparison of model results vs. real recordings

In Tropeano, G. et al. (2011) the model for differential displacements of two points on the soil surface, equation (9), excerpted from the Italian seismic code, has been compared with the results drawn from a subset of the Italian seismic databases Sisma and Itaca. Four real cases have been examined. Each case contains one couple of recording stations and many an event. The four couples of recording stations were on different or equal soil types, at distances ranging from about 200 to 600 meters. More precisely, the four selected cases were respectively relative to:

- subsoil couples (A, B), (A, C), (A, D), (B, B), according to the Seismic Italian Code 2008
- intensities, as measured by the peak ground acceleration, ranging from very low (0.006 g) to strong (0.35 g). A single very strong event (0.65 g) was also selected
- distances equal to 603 m, 435 m, 207 m, 431 m
- number of events equal to 8, 12, 20, 9.

The results (not shown here due to page limit) show an acceptable agreement with the model prediction, with the model always on the safe side. The model generally tends to strongly overestimate experimental results, for low intensities while it is in better agreement for higher intensities. However, this first comparison must be cautiously interpreted, because of the database paucity and because experimental recordings must be further post-processed (e.g. longitudinal and transverse displacement should be conveniently combined).

A further important remark concerns a conceptual point: the model (i.e. eq. (7)) gives the probability distribution of the maxima of a specific process, whereas the comparison is made against the maxima of single recordings. The comparisons are shown in Figure 6. Figure 6 shows the results for the four cases. Notice that intensities are generally low, with the exception of a single very strong shaking (0.65g, B-B soil coupling).

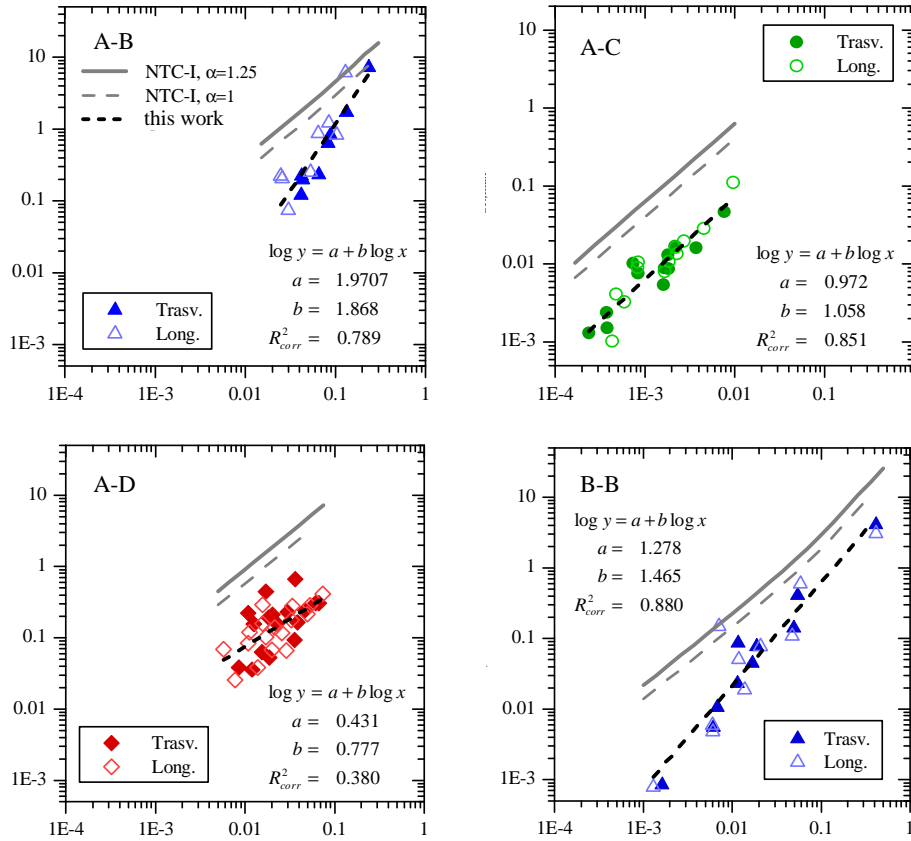


Figure 6. comparison between model and real recording differential displacements. Soil coupling A-B(top, left), A-C (top, right), A-D (bottom, left), B-B (bottom, right). p.g.a. (g) on the x-axis; maximum differential displacement (in 10-2 m) on the y-axis. Equation (9) predictions are in gray dotted line.

6. CONCLUSIONS

A theoretical model founded on basic random vibration theory has been developed in [Nuti, C. and Vanzi, I. (2004) & (2005)] and some of the results of the structural analyses based on this model are shown here. The model is here used to compute the differential displacements of points on the grounds, both for two and multiple points cases, considering both different code provisions (EC8 and new Italian Seismic Code) and contiguous different soils.

The results indicate that the design codes can be improved on this topic, both for the two points (e.g. simply supported decks) and the multiple points (e.g. continuous decks on multiple piers) cases, with the possible exception of the Italian Code for Bridges. For instance, for differential displacement for bridges, Eurocode 8 appears improvable, especially in the range of distances where most civil engineering structures are, below 100 m.

As for the structural response of statically undetermined structures under non synchronous action is concerned (continuous deck bridges are considered) it appears that, for the deck response variables, response spectrum values underestimate the target results. For the pier response variable, response spectrum analyses generally overestimate the target results.

The paper shows also the results of a preliminary experimental validation (model findings for the two points on the soil case, see paragraph 3 of this paper, vs. real recordings differential displacements). These show an acceptable agreement. The model generally tends to overestimate experimental results, for low intensities and it is in better agreement for higher intensities. However, this first comparison must be cautiously interpreted, mainly because of the database paucity.

ACKNOWLEDGEMENTS

The Authors thank for the support to their researches the Italian University Ministry (Prin 2008 Research Program; prot. n. 20083FFYWP; <http://prin.miur.it>).

REFERENCES

- Abrahamson, N. A., Schneider, J. F. and Stepp, J. C. (1991). Empirical spatial coherency functions for application to soil-structure interaction analyses. *Earthquake Spectra* **7:1**, 1-27.
- Biondi, S., Nuti, C. and Vanzi, I. (2011). Design actions for Continuous Deck Bridges considering Non Synchronous earthquake Motion. *5th Icege International Conference on Earthquake Geotechnical Engineering*, Dvd paper DAFNU, Santiago, Chile
- Carnevale, L., Lavorato, D., Nuti, C. and Vanzi, I. (2010). Response of continuous deck bridges to non synchronous seismic motion. *Proceedings of Sustainable Development Strategies for Constructions in Europe and China*, **Isbn 978-88-548-4418-6**: 441-450. Aracne Editrice 2012, Rome, Italy
- Clough, R. W. and Penzien, J. (1975). Dynamics of structures, McGraw-Hill Inc., New York, Usa.
- Comité Européen de Normalisation (2004). Eurocode 8: design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings, CEN, Bruxelles, Belgium
- Der Kiureghian, A. and Neuenhofer, A. (1991). A response spectrum method for multiple-support seismic excitations, UCB/EERC-91/08, Earthquake Engineering Research Center, University of California, Berkeley
- Der Kiureghian, A. and Neuenhofer, A. (1992). Response spectrum method for multi-support seismic excitations. *Earthquake Engineering & Structural Dynamics* **21:8**, 713-740.
- Hao, H. (1998). A parametric study of the required seating length for bridge decks during earthquake. *Earthquake Engineering & Structural Dynamics* **27:1**, 91-103.
- Luco, J. E. and Wong, H. L. (1986). Response of a rigid foundation to a spatially random ground motion. *Earthquake Engineering & Structural Dynamics* **14:6**, 891-908.
- Luco, J. E. and Mita, A. (1987). Response of circular foundation to spatially random ground motion. *Journal of Engineering Mechanics* **113:1**, 1-15.
- Ministero Infrastrutture (2008). Norme Tecniche per le Costruzioni. *Gazzetta Ufficiale*. **29:(S.O. 30)** (in Italian)
- Monti, G., Nuti, C., Pinto, P.E. and Vanzi, I. (1994). Effects of non Synchronous Seismic Input on the Inelastic Response of Bridges. *II International Workshop on Seismic Design of Bridges*, Queenstown, New Zealand
- Monti, G., Nuti, C. and Pinto, P. E. (1996). Nonlinear response of bridges under multisupport excitation. *Journal of Structural Engineering* **122:10**, 1147-1159.
- Newmark, N. M. and Hall, W. J. (1982). Earthquake spectra and design, UCB/EERC-82/03, Earthquake Engineering Research Center, University of California, Berkeley
- Nuti, C. and Vanzi, I. (2004). Influence of earthquake spatial variability on the differential displacements of soil and single degree of freedom structures, Structure Department Report, DIS 1/2004 Third University of Rome
- Nuti, C. and Vanzi, I. (2005). Influence of earthquake spatial variability on differential soil displacements and sdf system response. *Earthquake Engineering and Structural Dynamics* **34:11**, 1353-1374.
- Nuti, C. (Coordinator), (2010). Effects of non synchronism on seismic bridge response, including local site amplification. Ministero dell'Istruzione, dell'Università e della Ricerca, Prin 2008, Area 08 Civil Engineering and Architecture, **Research Program No. 32**, <http://prin.miur.it>
- Oliveira, C. S., Hao, H. and Penzien, J. (1991). Ground motion modeling for multiple-input structural analysis. *Structural Safety* **10:1-3**, 79-93.
- Presidenza del Consiglio dei Ministri (2003). Primi elementi in materia di criteri generali per la classificazione sismica del territorio nazionale e di normative tecniche per le costruzioni in zona sismica. Ordinanza No. 3274. *Gazzetta Ufficiale*. **108:(S.O. 72)** (in Italian)
- Santa-Cruz, S., Heredia-Zavoni, E. and Harichandran, R. S. (2000). Low-frequency behavior of coherency for strong ground motions in Mexico City and Japan", *12th World Conference on Earthquake Engineering. Proceedings:Paper No. 0076*
- Sextos, A.G., Ptilakis K.D. and Kappos, A. J. (2003). Inelastic dynamic analysis of RC bridges accounting for spatial variability of ground motion, site effects and soil-structure interaction phenomena. Part 1: Methodology and analytical tools. Part 2: Parametric study. *Earthquake Engineering & Structural Dynamics* **32:4**, 607-652.
- Tropeano, G., Silvestri, F., Nuti, C. and Vanzi, I. (2011). Influenza della risposta sismica locale sul non sincronismo del moto in superficie: validazione empirica delle indicazioni normative. *XIV Convegno L'Ingegneria Sismica in Italia*. Dvd paper, Ed. Digilabs, Bari, Italy
- Uscinski, B. J. (1977). The elements of wave propagation in random media, Mc Graw-Hill, New York, Usa
- Vanmarcke, E. H. and Fenton, G. A. (1991). Conditioned simulation of local fields of earthquake ground motion. *Structural Safety* **10:1-3**, 247-264
- Vanmarcke, E.H., Fenton, G.A. and Heredia-Zavoni, E. (1999). SIMQKE-II, conditioned earthquake ground motion simulator : user's manual version 2.1, Princeton University, Princeton, Usa